(F_y/3) in Equation (10-26) can be replaced by the allowable shearing stress given in Table 10.32.1A.

Transverse stiffeners shall be required if D/t_w is greater than 150. The spacing of these stiffeners shall not exceed the handling requirement $D\left[\dfrac{260}{D/t_w}\right]^2$.

10.34.4.3 The spacing of the first intermediate stiffener at the simple support end of a girder shall be such that the shearing stress in the end panel shall not exceed the value given by the following equation (the maximum spacing is limited to 1.5D):

$$F_v = \frac{C\,F_y}{3} \le \frac{F_y}{3} \qquad (10\text{-}29)$$

10.34.4.4 If a girder panel is subjected to simultaneous action of shear and bending moment with the magnitude of the shear stress higher than 0.6F_v, the calculated bending stress shall not exceed the reduced allowable bending stress, F_s determined by the following equation:

$$F_s = \left(0.754 - \frac{0.34 f_v}{F_v}\right) F_y \qquad (10\text{-}30)$$

where:

- f_v = average calculated shearing stress at the section; live load shall be the load to produce maximum moment at the section under consideration (psi)
- F_v = allowable shear stress obtained from Equation (10-26) (psi)
- F_s = reduced allowable bending stress (psi)

10.34.4.5 Where the calculated shear stress equals the allowable shear stress, transverse intermediate stiffeners may be omitted if the width-thickness ratio (D/t_w) of the web plate does not exceed the limiting values specified in Table 10.34.3A.

10.34.4.6 Intermediate stiffeners preferably shall be made of plates for welded plate girders and shall be made of angles for riveted plate girders. They may be in pairs, one stiffener fastened on each side of the web plate, with a tight fit at the compression flange. They may, however, be made of a single stiffener fastened to one side of the web plate. Stiffeners provided on only one side of the web must be welded to the compression flange and fitted tightly to the tension flange.

10.34.4.7 The width-thickness ratio (b'/t_s) of the transverse stiffener shall not exceed the limiting values specified in Table 10.34.5A. The moment of inertia of any type of transverse stiffener with reference to the plane defined in Article 10.34.4.8 shall meet the following requirement:

$$I \ge d_o\, t_w^3\, J \qquad (10\text{-}31)$$

where:

$$J = 2.5\left(\frac{D}{d_o}\right)^2 - 2 \ \ge 0.5 \qquad (10\text{-}32)$$

- I = minimum required moment of inertia of any type of transverse intermediate stiffener (in^4)
- J = ratio of rigidity of one transverse stiffener to that of the web plate
- d_o = spacing of transverse stiffeners (in.)
- D = unsupported depth of web plate between flange components (in.)
- t_w = thickness of the web plate (in.)

The gross cross-sectional area of intermediate transverse stiffeners, A (in.^{2}) shall meet the following requirement:

$$A \geq \left[0.15B\frac{D}{t_w}(1-C)\left(\frac{f_v}{F_v}\right)-18\right]\frac{F_{yweb}}{F_{cr}}t_w^2 \quad (10\text{-}32a)$$

where:

$$F_{cr} = \frac{9{,}025{,}000}{\left(\frac{b'}{t_s}\right)^2} \leq F_{ystiffener} \quad (10\text{-}32b)$$

b' = projecting width of the stiffener (in.)
t_s = thickness of the stiffener (in.)
F_{yweb} = specified minimum yield strength of the web (psi)
$F_{ystiffener}$ = specified minimum yield strength of the stiffener (psi)
B = 1.0 for stiffener pairs
1.8 for single angles and
2.4 for single plates
C = constant computed by Article 10.34.4.2.

When values computed by Equation (10-32a) approach zero or are negative, then transverse stiffeners need only meet the requirements of Equation (10-31), and the requirements of Article 10.34.4.10.

10.34.4.8 When stiffeners are in pairs, the moment of inertia shall be taken about the centerline of the web plate. When single stiffeners are used, the moment of inertia shall be taken about the face in contact with the web plate.

10.34.4.9 Transverse intermediate stiffeners shall be preferably fitted tightly to the tension flange. If the intermediate stiffener is used for attaching a cross frame or diaphragm, a positive connection using either bolts or welds must be made to the tension flange. The distance between the end of the vertical weld on the stiffener to the web-to-flange weld shall be $4t_w$ but not less than $1^1/_2$ inches. Stiffeners at points of concentrated loading shall be placed in pairs and should be designed in accordance with Article 10.34.6.

10.34.4.10 The width of a plate or the outstanding leg of an angle intermediate stiffener shall not be less than 2 inches plus $^1/_{30}$ the depth of the girder, and it shall preferably not be less than $^1/_4$ the full width of the girder flange. The thickness of a plate or the outstanding leg of an angle intermediate stiffener shall not be less than $^1/_{16}$ its width.

10.34.5 Longitudinal Stiffeners

10.34.5.1 The optimum distance, d_s, of a plate longitudinal stiffener or the gage line of an angle longitudinal stiffener from the inner surface or the leg of the compression flange component is $D/5$ for a symmetrical girder. The optimum distance, d_s, for an unsymmetrical composite girder in positive-moment regions may be determined from the equation given below:

$$\frac{d_s}{D_{cs}} = \frac{1}{1+1.5\sqrt{\frac{f_{DL+LL}}{f_{DL}}}} \quad (10\text{-}32c)$$

where:

D_{cs} = depth of the web in compression of the non-composite steel beam or girder (in.)
f_{DL} = non-composite dead-load stress in the compression flange (psi)
f_{DL+LL} = total non-composite and composite dead load plus the composite live-load stress in compression flange at the most highly stressed section of the web (psi)

The optimum distance, d_s, of the stiffener in negative-moment regions of composite sections is $2\,D_c/5$, where D_c is the depth of the web in compression of the composite section at the most highly stressed section of the web.

The longitudinal stiffener shall be proportioned so that:

$$I = D\,t_w^3\left(2.4\frac{d_o^2}{D^2}-0.13\right) \quad (10\text{-}33)$$

where:

I = required moment of inertia of the longitudinal stiffener about its edge in contact with the web plate (in.^{4})

D = unsupported distance between flange components (in.)
t_w = thickness of the web plate (in.)
d_o = spacing of transverse stiffeners (in.)

10.34.5.2 The width-thickness ratio (b'/t_s) of the longitudinal stiffener shall not exceed the limiting values specified in Table 10.34.5A.

10.34.5.3 The stress in the stiffener shall not be greater than the basic allowable bending stress for the material used in the stiffener.

TABLE 10.34.5A Limiting Width-Thickness Ratios for Stiffeners

Description of Component	Limiting (b'/t_s)
Longitudinal and Transverse stiffeners	$\frac{2,600}{\sqrt{F_y}}$ (10-34)
Bearing stiffeners	$\frac{2,180}{\sqrt{F_y}}$ (10-35) & (10-36)
Compression flange stiffeners	$\frac{2,600}{\sqrt{F_y}}$ (10-88)

b' = width of stiffener plate or outstanding legs of angle stiffener (in.)
F_y = specified minimum yield strength of stiffener (psi)
t_s = thickness of stiffener plate or outstanding legs of angle stiffener (in.)

10.34.5.4 Longitudinal stiffeners are usually placed on one side only of the web plate. They shall preferably be continuous where required. The termination and intersection of the longitudinal stiffener with transverse attachments shall consider the effects of fatigue. The interrupted element shall maintain the same strength characteristics as an uninterrupted element.

10.34.5.5 For longitudinally stiffened girders, transverse stiffeners shall be spaced a distance, d_o, according to shear capacity as specified in Article 10.34.4.2, but not more than 1.5 times the web depth. The handling requirement given in Article 10.34.4.2 shall not apply to longitudinally stiffened girders. The spacing of the first transverse stiffener at the simple support end of a longitudinally stiffened girder shall be such that the shearing stress in the end panel does not exceed the value given in Article 10.34.4.3. The total web depth D shall be used in determining the shear capacity of longitudinally stiffened girders in Articles 10.34.4.2 and 10.34.4.3.

10.34.5.6 Transverse stiffeners for girder panels with longitudinal stiffeners shall be designed according to Article 10.34.4.7.

10.34.6 Bearing Stiffeners

10.34.6.1 Welded Girders

Over the end bearings of welded plate girders and over the intermediate bearings of continuous welded plate girders there shall be stiffeners. They shall extend as nearly as practicable to the outer edges of the flange plates. They shall be made of plates placed on both sides of the web plate. Bearing stiffeners shall be designed as columns, and their connection to the web shall be designed to transmit the entire end reaction to the bearings. For stiffeners consisting of two plates, the column section shall be assumed to comprise the two plates and a centrally located strip of the web plate whose width is equal to not more than 18 times its thickness. For stiffeners consisting of four or more plates, the column section shall be assumed to comprise the four or more plates and a centrally located strip of the web plate whose width is equal to that enclosed by the four or more plates plus a width of not more than 18 times the web plate thickness. (See Article 10.40 for Hybrid Girders.) The radius of gyration shall be computed about the axis through the centerline of the web plate. The stiffeners shall be ground to fit against the flange through which they receive their reaction, or attached to the flange by full penetration groove welds. Only the portions of the stiffeners outside the flange-to-web plate welds shall be considered effective in bearing. The width-thickness ratio (b'/t_s) of the bearing stiffener plates shall not exceed the limiting values specified in Table 10.34.5A.

The allowable compressive stress and the bearing pressure on the stiffeners shall not exceed the values specified in Article 10.32.

10.34.6.2 Riveted or Bolted Girders

Over the end bearings of riveted or bolted plate girders there shall be stiffener angles, the outstanding legs of which shall extend as nearly as practicable to the outer edge on the flange angle. Bearing stiffener angles shall be proportioned for bearing on the outstanding legs of flange angles, no allowance being made for the portions of the legs being fitted to the fillets of the flange angles. Bearing stiffeners shall be arranged, and their connections to the web shall be designed to transmit the entire end reaction to the bearings. They shall not be crimped.

The width-thickness ratio (b'/t_s) of the bearing stiffener angles shall not exceed the limiting values specified in Table 10.34.5A.

The allowable compressive stress and the bearing pressure on the stiffeners shall not exceed the values specified in Article 10.32.

10.35 TRUSSES

10.35.1 Perforated Cover Plates and Lacing Bars

The shearing force normal to the member in the planes of lacing or continuous perforated plates shall be assumed divided equally between all such parallel planes. The shearing force shall include that due to the weight of the member plus any other external force. For compression members, an additional shear force shall be added as obtained by the following formula:

$$V = \frac{P}{100}\left[\frac{100}{l/r+10} + \frac{(l/r)F_y}{3{,}300{,}000}\right] \qquad (10\text{-}37)$$

where:

V = normal shearing force (lb.)
P = allowable compressive axial load on members (lb.)
l = length of member (in.)
r = radius of gyration of section about the axis perpendicular to plane of lacing or perforated plate (in.)
F_y = specified minimum yield strength of type of steel being used (psi)

10.35.2 Compression Members

10.35.2.1 Compression members shall be so designed that the main elements of the section will be connected directly to the gusset plates, pins, or other members.

10.35.2.2 The center of gravity of a built-up section shall coincide as nearly as practicable with the center of the section. Preferably, segments shall be connected by solid webs or perforated cover plates.

10.35.2.3 The with-thickness ratio (b/t) of elements of compression members shall not exceed the limiting values specified in Table 10.35.2A.

10.35.2.4 *Deleted*

10.35.2.5 *Deleted*

10.35.2.6 *Deleted*

10.35.2.7 *Deleted*

10.35.2.8 *Deleted*

10.35.2.9 *Deleted*

10.35.2.10 *Deleted*

10.35.2.11 *Deleted*

TABLE 10.35.2A Limiting Width-Thickness Ratios for Compression Member Elements

Description of Component	Limiting (b/t)	When $f_a = 0.44\ F_y$	
		F_y (psi)	Limiting b/t
Plates supported on one side, outstanding legs of angles and perforated plates–for outstanding plates, outstanding legs of angles, and perforated plates at the perforations	$\frac{1{,}625}{\sqrt{f_a}} \le \begin{cases} 12 & \text{for main members} \\ 16 & \text{for secondary member} \end{cases}$ (10-38)	36,000 50,000 70,000 90,000 100,000	12 11 9 8 7.5
Plates supported on two edges or webs of main component segments –for members of box shape consisting of main plates, rolled sections, or made up component segments with cover plates	$\frac{4{,}000}{\sqrt{f_a}} \le 45$ (10-39)	36,000 50,000 70,000 90,000 100,000	32 27 23 20 19
Solid cover plates supported on two edges or webs connecting main members or segments–for members of H or box shapes consisting of solid cover plates or solid webs connecting main plates or segments	$\frac{5{,}000}{\sqrt{f_a}} \le 50$ (10-40)	36,000 50,000 70,000 90,000 100,000	40 34 28 25 24
Perforated cover plates supported on two edges–for members of box shapes consisting of perforated cover plates connecting main plates or segments, perforated cover plates supported on one side	$\frac{6{,}000}{\sqrt{f_a}} \le 55$ (10-41)	36,000 50,000 70,000 90,000 100,000	48 41 34 30 29

b = distance between points of support (in.).
f_a = calculated compressive stress in the component under consideration (psi)
F_y = specified minimum yield strength of the component under consideration (psi)
t = component plate thickness (in.)

Note: The point of support shall be the inner line of fasteners or fillet welds connecting the plate to the main segment. For plates butt welded to the flange edge of rolled segments the point of support may be taken as the weld whenever the ratio of outstanding flange width to flange thickness of the rolled segment is less than seven. Otherwise, point of support shall be the root of flange of rolled segment. Terminations of the butt welds are to be ground smooth.

10.36 COMBINED STRESSES

All members subjected to both axial compression and flexure shall be proportioned to satisfy the following requirements:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1.0 \qquad (10\text{-}42)$$

and

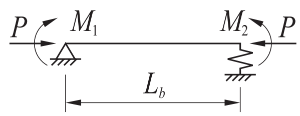
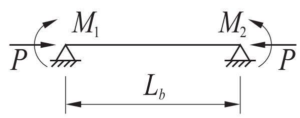
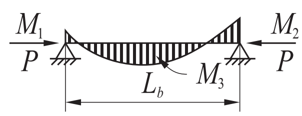
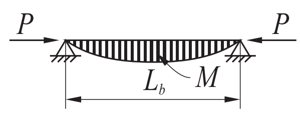
$$\frac{f_a}{0.472 F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \text{ (at points of support)} \qquad (10\text{-}43)$$

where:

$$F'_e = \frac{\pi^2 E}{F.S.\left(K_b L_b / r_b\right)^2} \qquad (10\text{-}44)$$

f_a = calculated axial stress (psi)
f_{bx}, f_{by} = calculated compressive bending stress about the x axis and y axis, respectively (psi)
F_a = allowable axial if axial force alone exists, regardless of the plane of bending (psi)
F_{bx}, F_{by} = allowable compressive bending stress if bending moment alone exists about the x axis and the y axis, respectively, as evaluated according to Table 10.32.1A (psi)
F'_e = Euler buckling stress divided by a factor of safety (psi)
E = modulus of elasticity of steel (psi)
K_b = effective length factor in the plane of bending (see Appendix C);
L_b = actual unbraced length in the plane of bending (in.)
r_b = radius of gyration in the plane of bending (in.)
C_{mx}, C_{my} = coefficient about the x axis and y axis, respectively, whose value is taken from Table 10.36A;
$F.S.$ = factor of safety = 2.12.

TABLE 10.36A Bending-Compression Interaction Coeffcients

Loading Conditions	Remarks	C_m
Computed moments maximum at end; joint translation not prevented		0.85
Computed moments maximum at end; no transverse loading, joint translation prevented		$\left[(0.4)\frac{M_1}{M_2}+0.6\right]$
Transverse loading; joint translation prevented		0.85
Transverse loading; joint translation prevented		1.0

M_1 = smaller end moment.
M_1/M_2 is positive when member is bent in single curvature.
M_1/M_2 is negative when member is bent in reverse curvature.
In all cases C_m may be conservatively taken equal to 1.0.

10.37 SOLID RIB ARCHES

10.37.1 Moment Amplification and Allowable Stress

10.37.1.1 The calculated compressive bending stress due to live load plus impact loading that are determined by an analysis which neglects arch rib deflection shall be increased by an amplification factor A_F:

$$A_F = \frac{1}{1 - \frac{1.7T}{AF_e}} \qquad (10\text{-}45)$$

where:

T = arch rib thrust at the quarter point from dead plus live plus impact loading (lb.)
F_e = Euler buckling stress (psi)

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad (10\text{-}46)$$

L = one half of the length of the arch rib (in)
A = area of cross section (in.^{2})
r = radius of gyration (in.)
K = effective length factor of the arch rib

K Values for Use in Calculating F_e and F_a

Rise to Span Ratio	3-Hinged Arch	2-Hinged Arch	Fixed Arch
0.1 - 0.2	1.16	1.04	0.70
0.2 - 0.3	1.13	1.10	0.70
0.3 - 0.4	1.16	1.16	0.72

10.37.1.2 The arch rib shall be proportioned to satisfy the following requirement:

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \leq 1.0 \qquad (10\text{-}47)$$

where:

f_a = the calculated axial stress (psi)
f_b = the calculated bending stress, including moment amplification, at the extreme fiber (psi)
F_a = the allowable axial stress (psi)
F_b = the allowable bending stress (psi)

10.37.1.3 For buckling in the vertical plane:

$$F_a = \frac{F_y}{F.S.}\left[1 - \frac{\left(\frac{K\,L}{r}\right)^2 F_y}{4\pi^2 E}\right] \qquad (10\text{-}48)$$

where KL as defined above and $F.S.$ is factor of safety = 2.12.

10.37.1.4 The effects of lateral slenderness should be investigated. Tied arch ribs, with the tie and roadway suspended from the rib, are not subject to moment amplification, and F_a shall be based on an effective length equal to the distance along the arch axis between suspenders, for buckling in the vertical plane. However, the smaller cross-sectional area of cable suspenders may result in an effective length slightly longer than the distance between suspenders.

10.37.2 Web Plates

10.37.2.1 The width-thickness ratio (D/t_w) of the web plates shall not exceed the limiting values specified in Table 10.37.2A.

10.37.2.2 If one longitudinal stiffener is used at mid-depth of the web, the moment of inertia of the stiffener about an axis parallel to the web and at the base of the stiffener shall meet the following requirement:

$$I_s \geq 0.75 D t_w^3 \qquad (10\text{-}51)$$

10.37.2.3 If two longitudinal stiffeners are used at the one-third points of the web depth D, the moment of inertia of each stiffener shall meet the following requirement:

$$I_s \geq 2.2 D t_w^3 \qquad (10\text{-}53)$$

10.37.2.4 The width-thickness ratio (b'/t_s) of any outstanding element of the web stiffeners shall not exceed the limiting values specified in Table 10.37.2A.

10.37.2.5 *Deleted*

10.37.3 Flange Plates

The width-thickness ratio (b'/t_f) of flange plates shall not exceed the limiting values specified in Table 10.37.2.A.

10.37.3.1 *Deleted*

10.37.3.2 *Deleted*

TABLE 10.37.2A Limiting Width-Thickness Ratios for Solid Rib Arches

Description of Component		Width Thickness Ratio	Limiting Width-Thickness Ratio	
Web Plates	Without longitudinal stiffiners	D/t_w	$\dfrac{5{,}000}{\sqrt{f_a}} \leq 60$	(10-49)
	With one longitudinal stiffener at the one-third point of the web depth		$\dfrac{7{,}500}{\sqrt{f_a}} \leq 90$	(10-50)
	With two longitudinal stiffeners at the one-third point of the web design		$\dfrac{10{,}000}{\sqrt{f_a}} \leq 120$	(10-52)
	Outstanding element of stiffiners	b'/t_s	$\dfrac{1{,}625}{\sqrt{f_a + f_b/3}} \leq 12$	(10-54)
Flange Plates	Web plate equations apply between limits		$0.2 \leq \dfrac{f_b}{\sqrt{f_a + f_b/3}} \leq 0.7$	(10-55)
	Plates between webs	b'/t_f	$\dfrac{4{,}250}{\sqrt{f_a + f_b}} \leq 47$	(10-56)
	Overhang plates		$\dfrac{1{,}625}{\sqrt{f_a + f_b}} \leq 12$	(10-57)

b' = width of flange plate or width of outstanding element of web stiffeners (in.)
f_a = calculated axial compressive stress in the component under consideration (psi)
f_b = calculated compressive bending stress in the component under consideration (psi)
t_f = flange plate thickness (in.)
t_s = web stiffener outstanding element thickness (in.)
t_w = web plate thickness (in.)

10.38 COMPOSITE BEAMS AND GIRDERS

10.38.1 General

10.38.1.1 This section pertains to structures composed of steel beams or girders with concrete slabs connected by shear connectors.

10.38.1.2 General specifications pertaining to the design of concrete and steel structures shall apply to structures utilizing composite girders where such specifications are applicable. Composite girders and slabs shall be designed and the stresses computed by the composite moment of inertia method and shall be consistent with the predetermined properties of the various materials used.

10.38.1.3 The ratio of the modulus of elasticity of steel (29,000,000 psi) to those of normal weight concrete (W = 145 pcf) of various design strengths shall be as follows:

f'_c = specified compressive strength of concrete as determined by cylinder tests at the age of 28 days (psi)

n = ratio of modulus of elasticity of steel to that of concrete. The value of n, as a function of the specified compressive strength of concrete, shall be assumed as follows:

f'_c		
=	2,000 - 2,300	$n = 11$
	2,400 - 2,800	$n = 10$
	2,900 - 3,500	$n = 9$
	3,600 - 4,500	$n = 8$
	4,600 - 5,900	$n = 7$
	6,000 or more	$n = 6$

10.38.1.4 The effect of creep shall be considered in the design of composite girders which have dead loads acting on the composite section. In such structures, bending stresses and horizontal shears produced by dead loads acting on the composite section shall be computed for n as given above or for this value multiplied by 3, whichever gives the higher bending stresses and shears.

10.38.1.5 If concrete with expansive characteristics is used, composite design should be used with caution and provision must be made in the design to accommodate the expansion.

10.38.1.6 Composite sections in simple spans and the positive moment regions of continuous spans should preferably be proportioned so that the neutral axis lies below the top surface of the steel beam. Concrete on the tension side of the neutral axis shall not be considered in calculating resisting moments. In the negative moment regions of continuous spans, only the slab reinforcement can be considered to act compositely with the steel beams in calculating resisting moments. Mechanical anchorages shall be provided in the composite regions to develop stresses on the plane joining the concrete and the steel. Concrete on the tension side of the neutral axis may be considered in computing moments of inertia for deflection calculations, for determining stiffness used in calculating moments and shears, and for computing fatigue stress ranges and fatigue shear ranges as permitted under the provisions of Article10.3.1and 10.38.5.1.

10.38.1.7 The steel beams or girders, especially if not supported by intermediate falsework, shall be investigated for stability and strength for the loading applied during the time the concrete is in place and before it has hardened. The casting or placing sequence specified in the plans for the composite concrete deck shall be considered when calculating the moments and shears on the steel section. The maximum flange compression stress shall not exceed the value specified in Table 10.32.1A for partially supported or unsupported compression flanges multiplied by a factor of 1.4, but not exceed $0.55F_y$. The sum of the non-composite and composite dead-load shear stresses in the web shall not exceed the shear-buckling capacity of the web multiplied by a factor of 1.35, nor the allowable shear stress, as follows:

$$F_v = 0.45CF_y \le 0.33F_y \qquad \text{(10-57a)}$$

where:

C = constant specified in Article 10.34.4.2.

10.38.2 Shear Connectors

10.38.2.1 The mechanical means used at the junction of the girder and slab for the purpose of developing the shear resistance necessary to produce composite action shall conform to the specifications of the respective materials. The shear connectors shall be of types that permit a thorough compaction of the concrete in order to ensure that their entire surfaces are in contact with the concrete. They shall be capable of resisting both horizontal and vertical movement between the concrete and the steel.

10.38.2.2 The capacity of stud and channel shear connectors welded to the girders is given in Article 10.38.5. Channel shear connectors shall have at least 3/16-inch fillet welds placed along the heel and toe of the channel.

10.38.2.3 The clear depth of concrete cover over the tops of the shear connectors shall be not less than 2 inches. Shear connectors shall penetrate at least 2 inches above bottom of slab.

10.38.2.4 The clear distance between the edge of a girder flange and the edge of the shear connectors shall be not less than 1 inch. Adjacent stud shear connectors shall not be closer than 4 diameters center to center.

10.38.3 Effective Flange Width

10.38.3.1 In composite girder construction the assumed effective width of the slab as a T-beam flange shall not exceed the following:

(1) One-fourth of the span length of the girder.
(2) The distance center to center of girders.
(3) Twelve times the least thickness of the slab.

10.38.3.2 For girders having a flange on one side only, the effective flange width shall not exceed one-twelfth of the span length of the girder, or six times the thickness of the slab, or one-half the distance center to center of the next girder.

10.38.4 Stresses

10.38.4.1 Maximum compressive and tensile stresses in girders that are not provided with temporary supports during the placing of the permanent dead load shall be the sum of the stresses produced by the dead loads acting on the steel girders alone and the stresses produced by the superimposed loads acting on the composite girder. When girders are provided with effective intermediate supports that are kept in place until the concrete has attained 75 percent of its required 28-day strength, the dead and live load stresses shall be computed on the basis of the composite section.

10.38.4.2 A continuous composite bridge may be built with shear connectors either in the positive moment regions or throughout the length of the bridge. The positive moment regions may be designed with composite sections as in simple spans. Shear connectors shall be provided in the negative moment portion in which the reinforcement steel embedded in the concrete is considered a part of the composite section. In case the reinforcement steel embedded in the concrete is not used in computing section properties for negative moments, shear connectors need not be provided in these portions of the spans, but additional anchorage connectors shall be placed in the region of the point of dead load contra-flexure in accordance with Article 10.38.5.1.3. Shear connectors shall be provided in accordance with Article 10.38.5.

10.38.4.3 The minimum longitudinal reinforcement including the longitudinal distribution reinforcement must equal or exceed one percent of the cross-sectional area of the concrete slab whenever the longitudinal tensile stress in the concrete slab due to either the construction loads or the design loads exceeds f_t specified in Article 8.15.2.1.1. The area of the concrete slab shall be equal to the structural thickness times the entire width of the bridge deck. The required reinforcement shall be No. 6 bars or smaller spaced at not more than 12 inches. Two-thirds of this required reinforcement is to be placed in the top layer of slab. Placement of distribution steel as specified in Article 3.24.10 is waived.

10.38.4.4 When shear connectors are omitted from the negative moment region, the longitudinal reinforcement shall be extended into the positive moment region beyond the anchorage connectors at least 40 times the reinforcement diameter. For epoxy-coated bars, the length to be extended into the positive moment region beyond the anchorage connectors should be modified to comply with Article 8.25.2.3.

10.38.5 Shear

10.38.5.1 Horizontal Shear

The maximum pitch of shear connectors shall not exceed 24 inches except over the interior supports of continuous beams where wider spacing may be used to avoid placing connectors at locations of high stresses in the tension flange.

Resistance to horizontal shear shall be provided by mechanical shear connectors at the junction of the concrete slab and the steel girder. The shear connectors shall be mechanical devices placed transversely across the flange of the girder spaced at regular or variable intervals. The shear connectors shall be designed for fatigue* and checked for design strength.

10.38.5.1.1 Fatigue

The range of horizontal shear shall be computed by the formula:

$$S_r = \frac{V_r Q}{I} \qquad (10\text{-}58)$$

where:

S_r = range of horizontal shear (lb/in.), at the junction of the slab and girder at the point in the span under consideration

V_r = range of shear due to live loads and impact (lb.); at any section, the range of shear shall be taken as the difference in the minimum and maximum shear envelopes (excluding dead loads)

Q = statical moment about the neutral axis of the composite section of the transformed concrete area (in^3). Between points of dead-load contraflexure, the static moment about the neutral axis of the composite section of the area of reinforcement embedded in the concrete may be used unless the transformed concrete area is considered to be fully effective for the negative moment in computing the longitudinal ranges of stress.

I = moment of inertia of the transformed short-term composite section (in^4). Between points of dead-load contraflexure, the moment of inertia of the steel girder including the area of reinforcement embedded in the concrete may be used unless the transformed concrete area is considered to be fully effective for the negative moment in computing the longitudinal ranges of stress.

(In the formula, the concrete area is transformed into an equivalent area of steel by dividing the effective concrete flange width by the modular ratio, *n*.)

* Reference is made to the paper titled "Fatigue Strength of Shear Connectors," by Roger G. Slutter and John W. Fisher, in *Highway Research Record*, No. 147, published by the Highway Research Board, Washington, D.C., 1966.

The allowable range of horizontal shear, Z_r (lb.) on an individual connector is as follows:

Channels:

$$Z_r = Bw \qquad (10\text{-}59)$$

Welded studs (for $H/d \geq 4$):

$$Z_r = \alpha d^2 \qquad (10\text{-}60)$$

where:

w = length of a channel shear connector (in.), measured in a transverse direction on the flange of a girder

d = diameter of stud (in.)

α = 13,000 for 100,000 cycles
10,600 for 500,000 cycles
7,850 for 2,000,000 cycles
5,500 for over 2,000,000 cycles;

B = 4,000 for 100,000 cycles
3,000 for 500,000 cycles
2,400 for 2,000,000 cycles
2,100 for over 2,000,000 cycles;

H = height of stud (in.).

The required pitch of shear connectors is determined by dividing the allowable range of horizontal shear of all connectors at one transverse girder cross-section (ΣZ_r) by the horizontal range of shear S_r, but not to exceed the maximum pitch specified in Article 10.38.5.1. Over the interior supports of continuous beams the pitch may be modified to avoid placing the connectors at locations of high stresses in the tension flange provided that the total number of connectors remains unchanged.

10.38.5.1.2 Design Strength

The number of connectors so provided for fatigue shall be checked to ensure that adequate connectors are provided for design strength.

The number of shear connectors required shall meet the following requirement:

$$N_1 \geq \frac{P}{\phi S_u} \qquad (10\text{-}61)$$

where:

N_1 = number of connectors between points of maximum positive moment and adjacent end supports
S_u = design strength of the shear connector as given below (lb.)
ϕ = reduction factor = 0.85;
P = horizontal shear force transferred by shear connectors as defined hereafter as P_1 or P_2.

At points of maximum positive moment, the force in the slab is taken as the smaller value of the formulas:

$$P_1 = A_s F_y \qquad (10\text{-}62)$$

or

$$P_2 = 0.85 f_c' b t_s \qquad (10\text{-}63)$$

where:

A_s = total area of the steel section including cover - plates (in.^{2})
F_y = specified minimum yield strength of the steel being used (psi)
f_c' = specified compressive strength of concrete at age of 28 days (psi)
b = effective flange width given in Article 10.38.3 (in.)
t_s = thickness of the concrete slab (in.)

The number of connectors, N_2, required between the points of maximum positive moment and points of adjacent maximum negative moment shall meet the following requirement:

$$N_2 \geq \frac{P + P_3}{\phi S_u} \qquad (10\text{-}64)$$

At points of maximum negative moment the force in the slab is taken as:

$$P_3 = A_s^r F_y^{r*} \qquad (10\text{-}65)$$

where:

A_s^r = total area of longitudinal reinforcing steel at the interior support within the effective flange width (in.^{2})
F_y^{r*} = specified minimum yield strength of the reinforcing steel (psi)

The design strength of the shear connector is given as follows:
Channels:

$$S_u = 550\left(h + \frac{t}{2}\right) W \sqrt{f_c'} \qquad (10\text{-}66)$$

Welded studs (for $H/d > 4$):

$$S_u = 0.4 d^2 \sqrt{f'_c E_c} \leq 60{,}000 A_{sc} \qquad (10\text{-}67)$$

where:

E_c = modulus of elasticity of the concrete (psi)

$$E_c = w^{3/2} 33 \sqrt{f_c'} \qquad (10\text{-}68)$$

S_u = design strength of individual shear connector (lb.)
h = average flange thickness of the channel flange (in.)
t = thickness of the web of a channel (in.)
W = length of a channel shear connector (in.)
f_c' = specified compressive strength of the concrete at 28 days (psi)
d = diameter of stud (in.)
w = unit weight of concrete (pcf)
A_{sc} = area of welded stud cross section (in.^{2})

10.38.5.1.3 Additional Connectors to Develop Slab Stresses

The number of additional connectors required at points of contraflexure when reinforcing steel embedded in the concrete is not used in computing section properties for negative moments shall be computed by the formula:

$$N_c = \frac{A_r^s f_r}{Z_r} \qquad (10\text{-}69)$$

where:

N_c = number of additional connectors for each beam at point of contraflexure

A_r^s = total area of longitudinal slab reinforcing steel for each beam over interior support (in.^{2})

f_r = range of stress due to live load plus impact in the slab reinforcement over the support (psi) (in lieu of more accurate computations, f_r may be taken as equal to 10,000 psi);

Z_r = allowable range of horizontal shear on an individual shear connector (lb.)

The additional connectors, N_c, shall be placed adjacent to the point of dead load contraflexure within a distance equal to one-third the effective slab width, i.e., placed either side of this point or centered about it. It is preferable to locate field splices so that they clear the connectors.

10.38.5.2 Vertical Shear

The intensity of shearing stress in a composite girder may be determined on the basis that the web of the steel girder carries the total external shear, neglecting the effects of the steel flanges and of the concrete slab. The shear may be assumed to be uniformly distributed throughout the gross area of the web.

10.38.6 Deflection

10.38.6.1 The provisions of Article 10.6 in regard to deflections from live load plus impact also shall be applicable to composite girders.

10.38.6.2 When the girders are not provided with falsework or other effective intermediate support during the placing of the concrete slab, the deflection due to the weight of the slab and other permanent dead loads added before the concrete has attained 75 percent of its required 28-day strength shall be computed on the basis of noncomposite action.

10.39 COMPOSITE BOX GIRDERS

10.39.1 General

10.39.1.1 This section pertains to the design of simple and continuous bridges of moderate length supported by two or more single cell composite box girders. The distance center-to-center of flanges of each box should be the same and the average distance center-to-center of flanges of adjacent boxes shall be not greater than 1.2 times and not less than 0.8 times the distance center-to-center of flanges of each box. In addition to the above, when nonparallel girders are used, the distance center-to-center of adjacent flanges at supports shall be no greater than 1.35 times and not less than 0.65 times the distance center-to-center of flanges of each box. The cantilever overhang of the deck slab, including curbs and parapets, shall be limited to 60 percent of the average distance center-to-center of flanges of adjacent boxes, but shall in no case exceed 6 feet.

10.39.1.2 The provisions of these Specifications shall govern where applicable, except as specifically modified by Articles 10.39.1 through 10.39.8.

10.39.2 Lateral Distribution of Loads for Bending Moment

10.39.2.1 The live load bending moment for each box girder shall be determined by applying to the girder, the fraction W_L of a wheel load (both front and rear), determined by the following equation:

$$W_L = 0.1 + 1.7R + \frac{0.85}{N_w} \qquad (10\text{-}70)$$

where:

$$R = \frac{N_w}{\text{Number of Box Girders}} \qquad (10\text{-}71)$$

N_w = W_c/12 reduced to the nearest whole number
W_c = roadway width between curbs (ft.), or barriers if curbs are not used.

R shall not be less than 0.5 or greater than 1.5.

10.39.2.2 The provision of Article 3.12, Reduction of Load Intensity, shall not apply in the design of box girders when using the design load W_L given by the above equation.

10.39.3 Web Plates

10.39.3.1 Vertical Shear

The design shear V_w for a web shall be calculated using the following equation:

$$V_w = \frac{V_v}{\cos\theta} \tag{10-72}$$

where:

V_v = vertical shear (lb.)
θ = angle of inclination of the web plate to the vertical.

10.39.3.2 Secondary Bending Stresses

10.39.3.2.1 Web plates may be plumb (90° to bottom of flange) or inclined. If the inclination of the web plates to a plane normal to bottom flange is no greater than 1 to 4, and the width of the bottom flange is no greater than 20 percent of the span, the transverse bending stresses resulting from distortion of the span, and the transverse bending stresses resulting from distortion of the girder cross section and from vibrations of the bottom plate need not be considered. For structures in this category transverse bending stresses due to supplementary loadings, such as utilities, shall not exceed 5,000 psi.

10.39.3.2.2 For structures exceeding these limits, a detailed evaluation of the transverse bending stresses due to all causes shall be made. These stresses shall be limited to a maximum stress or range of stress of 20,000 psi.

10.39.4 Bottom Flange Plates

10.39.4.1 General

The tension flange and the compression flange shall be considered completely effective in resisting bending if its width does not exceed one-fifth the span length. If the flange plate width exceeds one-fifth of the span, an amount equal to one-fifth of the span only shall be considered effective. Effective flange plate width shall be used to calculate the flange bending stress. Full flange plate width shall be used to calculate the allowable compressive bending stress.

10.39.4.1.2 Deleted

10.39.4.2 Compression Flanges Unstiffened

10.39.4.2.1 For unstiffened compression flanges, the calculated bending stress shall not exceed the allowable bending stress, F_b (psi), determined by either of the following equations:

for $\dfrac{b}{t} \le \dfrac{6{,}140}{\sqrt{F_y}}$

$$F_b = 0.55F_y \tag{10-73}$$

for $\dfrac{6{,}140}{\sqrt{F_y}} < \dfrac{b}{t} \le \dfrac{13{,}300}{\sqrt{F_y}}$

$$F_b = 0.55F_y - 0.224F_y\left[1 - \sin\left(\frac{\pi}{2}\,\frac{13{,}300 - \dfrac{b\sqrt{F_y}}{t}}{7{,}160}\right)\right] \tag{10-74}$$

for $\dfrac{b}{t} \ge \dfrac{13{,}300}{\sqrt{F_y}}$

$$F_b = 57.6\left(\frac{t}{b}\right)^2 \times 10^6 \tag{10-75}$$

where:

b = flange width between webs (in.)
t = flange thickness (in.)

10.39.4.2.2 Deleted

10.39.4.2.3 Deleted

10.39.4.2.4 The b/t ratio preferably should not exceed 60 except in areas of low stress near points of dead load contraflexure.

10.39.4.2.5 If the b/t ratio exceeds 45, longitudinal stiffeners may be considered.

10.39.4.2.6 Deleted

10.39.4.3 Compression Flanges Stiffened Longitudinally*

10.39.4.3.1 Longitudinal stiffeners shall be at equal spacings across the flange width and shall be proportioned so that the moment of inertia of each stiffener about an axis parallel to the flange and at the base of the stiffener, I_s (in.^{4}) shall meet the following requirement:

$$I_s \geq \phi t_f^3 w \qquad (10\text{-}76)$$

where:

ϕ = 0.07 $k^3 n^4$ for values of n greater than 1;
ϕ = 0.125 k^3 for a value of n = 1;
t_f = thickness of the flange (in.)
w = width of flange between longitudinal stiffeners or distance from a web to the nearest longitudinal stiffener (in.)
n = number of longitudinal stiffeners;
k = buckling coefficient which shall not exceed 4.

10.39.4.3.2 For the longitudinally stiffened flange, including stiffeners, the calculated bending stress shall not exceed the allowable bending stress, F_b (psi), determined by either of the following equations:

for $$\frac{w}{t} \leq \frac{3{,}070\sqrt{k}}{\sqrt{F_y}}$$

$$F_b = 0.55F_y \qquad (10\text{-}77)$$

for $$\frac{3{,}070\sqrt{k}}{\sqrt{F_y}} < \frac{w}{t} \leq \text{smaller of } \{60\} \text{or} \left\{\frac{6{,}650\sqrt{k}}{\sqrt{F_y}}\right\}$$

$$F_b = 0.55F_y - 0.224F_y\left[1 - \sin\left(\frac{\pi}{2}\,\frac{6{,}650\sqrt{k} - \dfrac{w\sqrt{F_y}}{t}}{3{,}580\sqrt{k}}\right)\right] \qquad (10\text{-}78)$$

for $$\frac{6{,}650\sqrt{k}}{\sqrt{F_y}} \leq \frac{w}{t} \leq 60$$

$$F_b = 14.4k\left(\frac{t}{w}\right)^2 \times 10^6 \qquad (10\text{-}79)$$

10.39.4.3.3 Deleted

10.39.4.3.4 Deleted

10.39.4.3.5 When longitudinal stiffeners are used, it is preferable to have at least one transverse stiffener placed near the point of dead load contraflexure. The stiffener should have a size equal to that of a longitudinal stiffener.

10.39.4.3.6 If the longitudinal stiffeners are placed at their maximum w/t ratio to be designed for the basic allowable design stresses of $0.55F_y$ and the number of longitudinal stiffeners exceeds 2, then transverse stiffeners should be considered.

10.39.4.3.7 Deleted

* In solving these equations a value of k between 2 and 4 generally should be assumed.

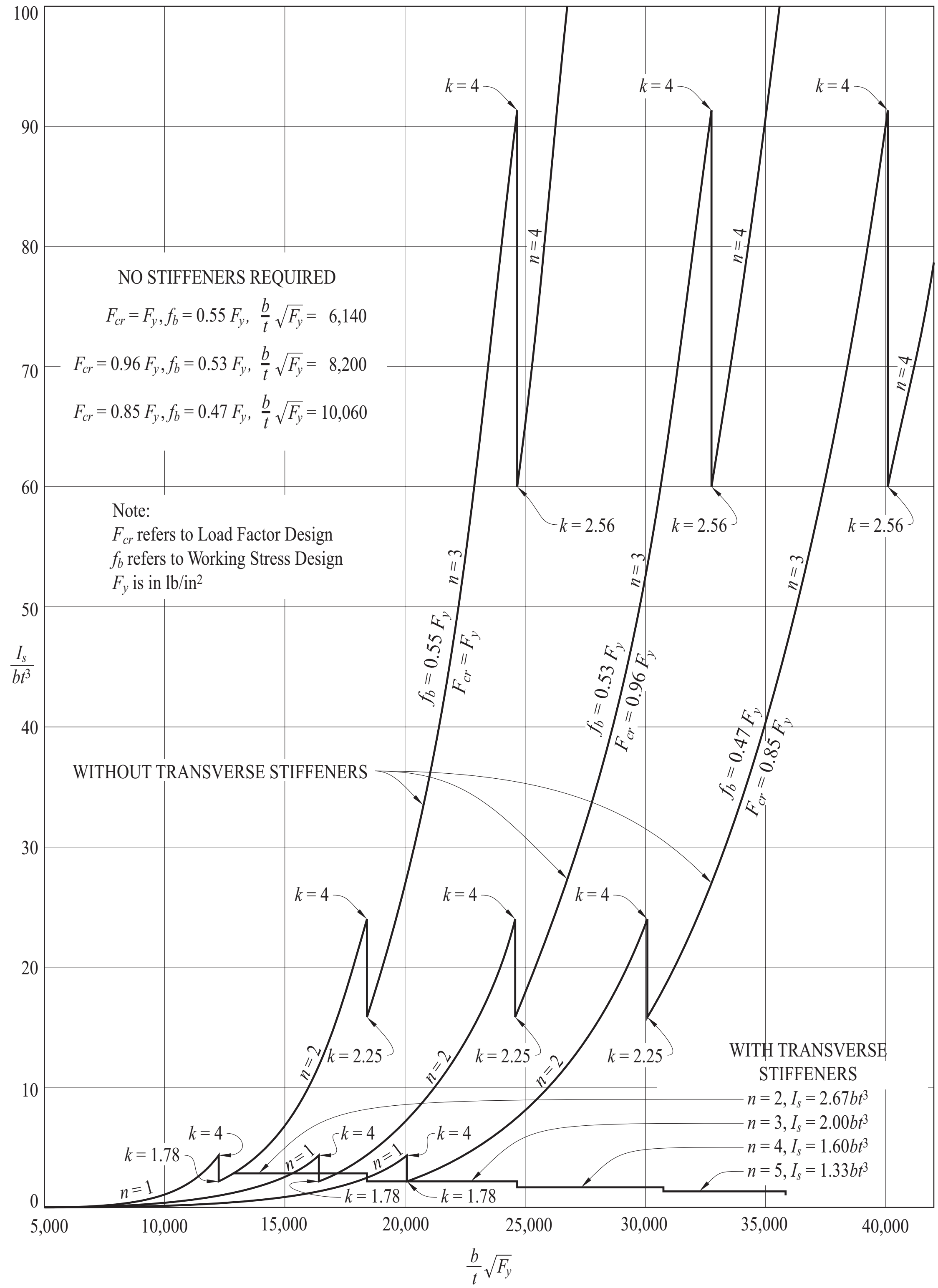


FIGURE 10.39.4.3A Longitudinal StiffenersñBox Girder Compression Flange

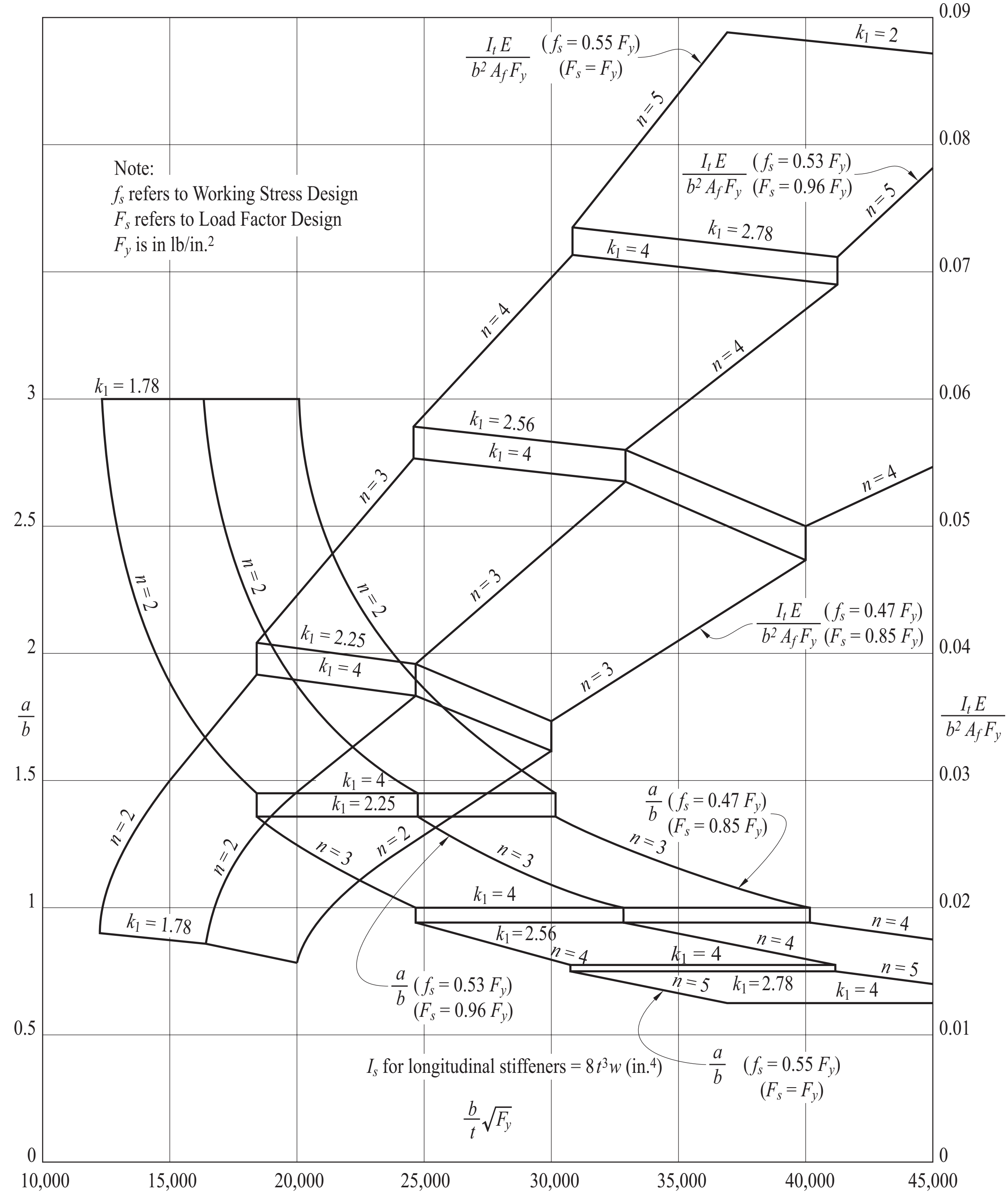


FIGURE 10.39.4.3B Spacing and Size of Transverse Stiffeners (for Flange Stiffened Longitudinally and Transversely)

10.39.4.4 Compression Flanges Stiffened Longitudinally and Transversely

10.39.4.4.1 The longitudinal stiffeners shall be at equal spacings across the flange width and shall be proportioned so that the moment of inertia of each stiffeners about an axis parallel to the flange and at the base of the stiffeners meet the following requirement:

$$I_s \geq 8 t_f^3 w \qquad (10\text{-}80)$$

10.39.4.4.2 The transverse stiffeners shall be proportioned so that the moment of inertia of each stiffener about an axis through the centroid of the section and parallel to its bottom edge meets the following requirement:

$$I_t \geq 0.10 \left(n+1\right)^3 w^3 \frac{f_s}{E} \frac{A_f}{d_o} \qquad (10\text{-}81)$$

where:

A_f = area of bottom flange including longitudinal stiffeners (in.^{2})
d_o = spacing of transverse stiffeners (in.)
f_s = maximum longitudinal bending stress in the flange of the panels on either side of the transverse stiffeners (psi)
E = modulus of elasticity of steel (psi)

10.39.4.4.3 For the flange, including stiffeners, the calculated bending stress shall not exceed the allowable bending stress, F_b (psi), determined by either of the following equations:

for $\dfrac{w}{t} \leq \dfrac{3{,}070\sqrt{k_1}}{\sqrt{F_y}}$

$$F_b = 0.55 F_y \qquad (10\text{-}82)$$

for $\dfrac{3{,}070\sqrt{k_1}}{\sqrt{F_y}} < \dfrac{w}{t} \leq$ smaller of $\{60\}$ or $\left\{\dfrac{6{,}650\sqrt{k_1}}{\sqrt{F_y}}\right\}$

$$F_b = 0.55 F_y - 0.224 F_y \left[1 - \sin\left(\frac{\pi}{2}\,\frac{6{,}650\sqrt{k_1} - \dfrac{w\sqrt{F_y}}{t}}{3{,}580\sqrt{k_1}}\right)\right] \qquad (10\text{-}83)$$

for $\dfrac{6{,}650\sqrt{k_1}}{\sqrt{F_y}} \leq \dfrac{w}{t} \leq 60$

$$F_b = 14.4\, k_1 \left(\frac{t}{w}\right)^2 \times 10^6 \qquad (10\text{-}84)$$

where:

$$k_1 = \frac{\left[1 + \left(a/b\right)^2\right]^2 + 87.3}{\left(n+1\right)^2 \left(a/b\right)^2 \left[1 + 0.1\left(n+1\right)\right]} \qquad (10\text{-}85)$$

10.39.4.4.4 Deleted

10.39.4.4.5 Deleted

10.39.4.4.6 The maximum value of the buckling coefficient, k_1, shall be 4. When k_1 has its maximum value, the transverse stiffeners shall have a spacing, a, equal to or less than $4w$. If the ratio a/b exceeds 3, transverse stiffeners are not necessary.

10.39.4.4.7 The transverse stiffeners need not be connected to the flange plate but shall be connected to the webs of the box and to each longitudinal stiffener. The connection to the web shall be designed to resist the vertical force determined by the formula:

$$R_w = \frac{F_y S_s}{2b} \qquad (10\text{-}86)$$

where:

S_s = section modulus of the transverse stiffener (in.^{3})

10.39.4.4.8 The connection to each longitudinal stiffener shall be designed to resist the vertical force determined by the formula:

$$R_s = \frac{F_y S_s}{n\,b} \qquad (10\text{-}87)$$

10.39.4.5 Compression Flange Stiffeners, General

+ *10.39.4.5.1* The width-thickness ratio (b'/t_s) of any outstanding element of the flange stiffeners shall not exceed the limiting values specified in Table 10.34.5A. +

10.39.4.5.2 Longitudinal stiffeners shall be extended to locations where the maximum stress in the flange does not exceed that allowed for base metal adjacent to or connected by fillet welds.

+ 10.39.5 Flange to Web Welds

The total effective thickness of the web-flange welds shall be not less than the thickness of the web. + Regardless of the type weld used, welds shall be deposited on both sides of the connecting flange or web plate.

10.39.6 Diaphragms

10.39.6.1 Diaphragms, cross-frames, or other means shall be provided within the box girders at each support to resist transverse rotation, displacement, and distortion.

10.39.6.2 Intermediate diaphragms or cross-frames are not required for steel box girder bridges designed in accordance with this specification.

10.39.7 Lateral Bracing

Generally, no lateral bracing system is required between box girders. A horizontal wind load of 50 pounds per square foot shall be applied to the area of the superstructure exposed in elevation. Half of the resulting force shall be applied in the plane of the bottom flange. The section assumed to resist the horizontal load shall consist of the bottom flange acting as a web and 12 times the thickness of the webs acting as flanges. A lateral bracing system shall be provided if the combined stresses due to the specified horizontal force and dead load of steel and deck exceed 150 percent of the allowable design stress.

10.39.8 Access and Drainage

Consistent with climate, location, and materials, consideration shall be given to the providing of manholes, or other openings, either in the deck slab or in the steel box for form removal, inspection, maintenance, drainage, etc.

10.40 HYBRID GIRDERS

10.40.1 General

10.40.1.1 This section pertains to the design of girders that utilize a lower strength steel in the web than in one or both of the flanges. It applies to composite and noncomposite plate girders, and composite box girders. At any cross section where the bending stress in either flange exceeds 55 percent of minimum specified yield strength of the web steel, the compression-flange area shall not be less than the tension-flange area. The top-flange area shall include the transformed area of any portion of the slab or reinforcing steel that is considered to act compositely with the steel girder.

10.40.1.2 The provisions of these Specifications, shall govern where applicable, except as specifically + modified by Articles 10.40.1 through 10.40.4.

10.40.2 Allowable Stresses

10.40.2.1 Bending

10.40.2.1.1 The bending stress in the web may exceed the allowable stress for the web steel provided that the stress in each flange does not exceed the allowable stress from Articles 10.3 or 10.32 for the steel in that flange multiplied by the reduction factor, *R*.

$$R = 1 - \frac{\beta\psi(1-\alpha)^2(3-\psi+\psi\alpha)}{6+\beta\psi(3-\psi)} \qquad (10\text{-}89)$$

(See Figures 10.40.2.1A and 10.40.2.1B)

where:

α = specified minimum yield strength of the web divided by the specified minimum yield strength + of the tension flange;*

β = area of the web divided by the area of the tension flange;*

* Bottom flange of orthotropic deck bridge